# Geotechnical Engineering Report
# Mt. Zion Old School Baptist Church
# Damage Study
# Gilberts Corner
# Loudoun County, Virginia

Schnabel Engineering

Schnabel Engineering

Schnabel Engineering Associates, Inc.
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December 9, 1997

Mr. Baird M. Smith, AIA
Quinn Evans/Architects
1214 Twenty-Eighth Street, N.W.
Washington, D.C. 20007

Subject: **Geotechnical Engineering Report, Mt. Zion Old School Baptist Church Damage Study, Gilberts Corner, Loudoun County, Virginia (Our 972717)**

Dear Mr. Smith:

Schnabel Engineering Associates, Inc. is pleased to submit this geotechnical engineering report for the above referenced project. This study was performed in accordance with the scope of services identified in the technical approach of the request for proposal from Loudoun County dated June 10, 1997 and your contract dated October 17, 1997.

## 1.0 Scope of Services

This study includes the results of a field investigation, soil laboratory testing, and engineering analysis of the test data and project information to develop a geotechnical engineering report. This report specifically addresses the following:

1. Evaluation of estimated subsurface conditions at the subject property as made evident from the test pit investigation.

2. Recommendations regarding existing foundation configurations and bearing pressures, based on information obtained from test pit observations.

3. Evaluation of the probable cause of settlement observed at the rear of the subject property, and the potential for future settlements in this area. Comments regarding possible remedial measures considered necessary to arrest further movement are also provided.

Our scope does not include services with respect to specific construction dewatering recommendations, environmental matters, paving design, erosion control, cost or quantity estimates, plans, specifications, and construction observation and testing.

## 2.0 Background Information and Site Observations

The Mt. Zion Old School Baptist Church is located on the south side of U.S. Route 50 (Lee-Jackson Memorial Highway) approximately 0.8 miles east of its intersection with U.S. Route 15 (James Madison Highway) in Gilberts Corner, Loudoun County, Virginia. The Mt. Zion Church was constructed in 1851 and is a two-story, gable roofed, brick building with a stone foundation. The church building is approximately 46 feet by 36 feet in plan.

Structural damage was observed on the west wall of the church consisting of an apparent tilt towards the west and cracking of the west end of the north and south walls. Raking shores had been installed on the west wall at the time of our site visits.

## 3.0 Subsurface Conditions

A field and laboratory investigation program was completed. This program consisted of excavating and logging three shallow test pits, dynamic cone penetrometer testing of footing subgrade soils, and soil laboratory testing of selected samples collected during the test pit investigation. The test pit logs are presented in Appendix A. Test pit locations are presented on Figure 1 in Appendix A. Soil test pits were located by the project archeological team. Test pit Nos. AU-2 and AU-4 were excavated by the archeologist to a level equal to the base of the foundation. Test pit No. AU-3 was excavated by the archeologist to a depth of approximately 1.5 feet below the ground surface. Test pit No. AU-3 was excavated from this point to the bottom of the foundation by Schnabel Engineering Associates personnel. Excavations below the level of the bottom of the foundation were made by SEA personnel using a hand auger with a 3 inch diameter auger bucket. Photographs taken at the site during our subsurface investigation are presented in Appendix C.

### 3.1 Soil Characteristics

Excavation of three test pits were made adjacent to the rear, west side, and front exterior load bearing walls in order to evaluate subsurface conditions at foundation depths. Observations of the test pits indicated a small amount of surficial fill (in test pit AU-2 only) overlying medium to hard cohesive soils with rock fragments and sand to depths of 3.0 to 4.6 feet, the maximum depths investigated. The fill is designated in this report as Stratum A and underlying natural soils as Stratum B. The natural soils of Stratum B were found to be relatively plastic and are likely subject to shrink/swell with variations in moisture content.

### 3.2 In Situ Test Results

In order to assess the bearing capacity of soils encountered at foundation subgrade depths, dynamic cone penetration (DCP) tests were completed in each test pit. A description of the DCP test procedures is presented in the subsurface investigation procedures in Appendix A.

The DCP test results indicated N-value blow counts of from 5 to greater than 50. Geostick penetrations of the soil at foundation depths indicated penetrations of less than approximately 1 inch. The general consistency of the foundation subgrade soils at the test pit locations were found to be medium to hard.

### 3.3 Geology

The Mt. Zion Old School Baptist Church site lies entirely within the Culpepper Basin of the Piedmont Physiographic province. The Culpepper Basin is a rift related basin, or graben, which formed from the tensional forces created by continental rifting during the Mesozoic Geologic Era. During the Mesozoic era, this basin filled with lacustrine and alluvial fan sediments which were intruded periodically by diabase and basalt dikes, sills, and flows. These sediments were subsequently lithified into shales, siltstones, sandstones, and conglomerates.

The soils observed in the test pits and those recovered from the hand auger borings have been stratified for purposes of our discussion herein. These stratum designations do not imply continuity of the materials described, but the general description and characteristics of the materials at the site.

The fill soils of Stratum A appear to be grading fill placed some time after construction to increase the slope of the ground surface away from the church structure. The natural soils of Stratum B are residual materials derived from the in-place weathering of the underlying parent bedrock. The bedrock beneath the school site is believed to be a basalt rock belonging to the Mt. Zion Church Formation from Lower Jurassic Geologic Period. This bedrock consists of dark gray to black very-fine crystalline vesicular basalt. This bedrock often forms north-south trending linear ridges which weather into a soil profile consisting of a thin layer of cohesive soils underlain by disintegrated rock, and bedrock. This bedrock is severely fractured and jointed, and as a result, the disintegrated rock portion of the subsurface profile often contains unweathered cobbles and boulders of basalt in a soil matrix. These "floaters" are often mistaken to be bedrock.

### 3.4 Ground Water

No ground water was noted in the soil test pits however, Test Pit No. AU-2 was found to be moist to wet after allowing the test pit to remain open over night. The wet conditions may be related to moisture which has collected below the building and around the foundations. Based on these observations, we estimate the ground water levels are below the levels of our test pits. However, ground water levels can vary depending on seasonal variations, precipitation, surface runoff and similar factors.

### 3.5 Soil Laboratory Test Results

Selected soil samples recovered from the test pits were tested in our laboratory to evaluate natural moisture content, classification, and Atterberg Limits. The laboratory test results are presented in Appendix B.

A bag sample from Test Pit No. AU-2 at a depth of 3.0 feet consisted of FAT CLAY with 77.5percent fines passing the U.S. Standard No. 200 sieve. This sample had a liquid limit of 71 and a plasticity index of 38. This sample classified as CH per ASTM D-2487.

A bag sample from Test Pit No. AU-2 at a depth of 3.6 feet consisted of sandy LEAN CLAY with 55.6 percent fines passing the U.S. Standard No. 200 sieve. This sample had a liquid limit of 46 and a plasticity index of 19. This sample classified as CL per ASTM D-2487.

A bag sample from Test Pit No. AU-3 at a depth of 2.5 feet consisted of ELASTIC SILT with 88.1 percent fines passing the U.S. Standard No. 200 sieve. This sample classified as MH per ASTM D-2487 and had a liquid limit of 58 and a plasticity index of 28.

A bag sample collected from Test Pit No. AU-4 at a depth of 2.8 feet consisted of ELASTIC SILT with 67.8 percent fines passing the U.S. Standard No. 200 sieve. This sample had a liquid limit of 78 and a plasticity index of 42 and classified as MH per ASTM D-2487.

Natural moisture content testing was also performed on selected bag samples. Natural moisture contents of the samples analyzed ranged from 24.4 to 36.9 percent.

## 4.0 Existing Conditions

The test pit excavations indicated that the existing stone foundation extends approximately 2.5 feet below the ground surface. The stone used to construct the foundation appears to be locally obtained basalt cobbles, which are approximately 6 to 12 inches in size. The stones appear to have been bonded together with mortar, however, the majority of the mortar has disintegrated over time. One large stone in the foundation wall at Test Pit No. AU-3 was found to be fractured. The aperture of this fracture was approximately 0.5 inches.

As part of our test pit observations, dynamic cone penetrometer testing of the soils present at foundation grades was performed to assess soil bearing capacities. Based on these results, the soil present at, and below foundation grades, are suitable for an allowable bearing pressure of 3,000 psf.

## 5.0 Probable Causes of the Distress

The structural damage observed on the west end of the church building can be attributed to two geotechnical issues; settlement of the foundation due to softening of the subsoils, or foundation heaving due to shrink/swell soils. Our limited subsurface investigation indicated that the soils present at foundation grades are firm and are suitable for support of the church structure. Secondly, post construction settlement of this structure would have occurred during the first 5-10 years of the church's design life. Any recent settlement would have to be the result of a significant change in building conditions such as extreme wetting of foundation soils or increased loading. Our investigation did not indicate excessively wet soils and the structural

loads on this building have not increased.

The results of soil laboratory testing of soil samples collected during our subsurface investigation indicate that the soils present at, and below foundation grades, are relatively plastic. These soils are likely subject to volume changes with variations in moisture contents, however laboratory testing to determine the extent of the shrink/swell potential of these soils was beyond the scope of this project. In addition, based on discussions with Mr. Rick Ortega, structural engineer, the area where the most prominent damage was observed was where the structure is lightly loaded. If the foundation soils were subject to shrink/swell cycles, the lightly loaded areas would be affected the most. This condition appears to be the most likely geotechnical related contributor to the observed structural damage.

## 6.0 Recommended Remedial Measures

In order to arrest further settlement of the west wall foundation, the west wall foundation should be underpinned to a minimum depth of six feet below the ground surface or to below the level of moisture variation. Underpinning generally consists of excavating an area with a maximum width of five feet beneath the footing down to suitable bearing material and filling the resulting excavation with reinforced concrete. Underpinning of the stone foundation at the Mt. Zion site may be difficult because foundation stones may fall out from the foundation wall as the wall is undermined. Specific details of underpinning should be provided by a specialty contractor with experience in working with historical structures.

## 7.0 General

This report has been prepared for the exclusive use of the Client for specific application to this project. It is noted that the subsurface investigation was limited in scope. The findings and recommendations presented herein are based on the relatively limited number of test pits and information obtained. There may be other factors which are contributing to, or causing, the observed building damage that were not identified during the study. Schnabel Engineering Associates has endeavored to comply with generally accepted geotechnical engineering practice common to the local area, and make no warranty, expressed or implied as to the professional advice presented herein.

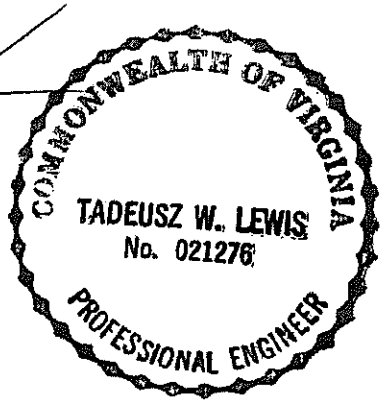
We appreciate the opportunity to be of service for this project. Please contact us for further clarification of any aspect of this report.

Very truly yours,

SCHNABEL ENGINEERING ASSOCIATES, INC.

R. Drew Thomas, P.G.
Senior Staff Geologist

Tadeusz W. Lewis, P.E.
Senior Associate

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Appendix A: Subsurface Investigation Report
Test Pit Logs
Test Pit Location Plan

Appendix B: Soil Laboratory Test Results

Appendix C: Site Photographs

Appendix A
Contract No. 972717

# SUBSURFACE INVESTIGATION REPORT

Subsurface Investigation Procedures
Test Pit Logs (3 Sheets)
Test Pit Location Plan

Appendix A
Contract No. 972717

# SUBSURFACE INVESTIGATION PROCEDURES

1. Test Pit Observations and General Notes

Test pits were logged by SEA personnel to provide a record for geotechnical evaluation construction inspection or other specialized purpose. The log includes a description of soil materials encountered using visual classification in the field. Boundary lines between various strata are identified where possible and a graphical presentation is included based on the material excavated from the pit. Any significant features, such as fill conditions, underground structures, ground water or water seepage conditions are recorded. Representative soil samples were recovered from the test pits, generally from each stratum, for later identification and testing. The locations of these samples are generally not shown on the logs, unless laboratory tests are performed on samples are referred to in the geotechnical analysis.

The test pit logs and related information depict subsurface conditions only at the specific location and at the particular time excavated. Soil conditions at other locations may differ from conditions occurring at these test pit locations. Also, the passage of time may result in a change in the subsurface soil and ground water conditions at these locations. The stratification lines represent the approximate boundary between soil types as observed in the test pits. The soil profiles and water level observations presented have been made with reasonable care and accuracy and must be considered only an approximate representation of subsurface conditions to be encountered at the particular location. Mr. R. Drew Thomas, Senior Staff Geologist from our office logged the test pits.

2. The dynamic cone penetration (DCP) test procedures

The dynamic cone penetration (DCP) test procedures we are currently using follows the method developed by "Sowers and Hedges" for this equipment as presented in Astm stp 399, 1966, P. 29.

The test procedure generally consists of advancing the hand auger to the proposed test depth and seating the cone 1- ½ to 2 inches into the soil. The cone is then driven in successive 1- ¾ inch intervals using a 15 pound hammer free falling 20 inches. The number of blows required to drive the cone each interval is recorded. The cone is normally driven for three intervals at each test depth. A sample of soil tested is then obtained by reinserting the hand auger.

**SCHNABEL ENGINEERING**

# TEST PIT LOG

| | | | | | |
|---|---|---|---|---|---|
| **CONTRACT NO.:** | 972717 | **DATE STARTED:** | 11/20/97 | **DATE ENDED:** | 11/20/97 |
| **EXCAVATION NO.:** | AU-2 | **SURFACE EL.:** | | **GROUNDWATER EL.:** | NONE ENCOUNTERED |
| **PROJECT:** | MT. ZION OLD SCHOOL BAPTIST SCHOOL | **EXCAVATION EQUIP.:** | HAND EXCAVATED | | |
| **LOCATION:** | GILBERTS CORNER, VA | **SEA REPRESENTATIVE:** | R. DREW THOMAS | | |

| DEPTH (ft) | ELEV. (ft) | DESCRIPTION OF SOILS AND OBSERVATIONS | STRATUM | DYNAMIC CONE PENETROMETER |
|---|---|---|---|---|
| 0.8 | | lean clay, trace sand and rock fragments, **FILL**, moist, reddish-brown | A | |
| 3.5 | | **FAT CLAY (CH)**, trace sand, rock fragments and organics, moist, brown | B | 4-5-5-6 @ 2.5 ft<br>8-15-35 @ 3.2 ft<br>35-50/0.5" @ 3.8 ft |
| 4.5 | | sandy **LEAN CLAY (CL)**, with rock fragments, moist, brown | | 37-50/1" @ 4.5 ft |
| | | Hand auger refusal @ 4.5 ft | | |

Comments: Bottom of stacked stone foundation @ 2.5 ft.

SCHNABEL ENGINEERING [redacted]

# TEST PIT LOG

CONTRACT NO.: 972717
EXCAVATION NO.: AU-3
PROJECT: MT. ZION OLD SCHOOL BAPTIST SCHOOL
LOCATION: GILBERTS CORNER, VA

DATE STARTED: 11/20/97 DATE ENDED: 11/20/97
SURFACE EL.:
GROUNDWATER EL.: NONE ENCOUNTERED
EXCAVATION EQUIP.: HAND EXCAVATED
SEA REPRESENTATIVE: R. DREW THOMAS

| DEPTH (ft) | ELEV. (ft) | DESCRIPTION OF SOILS AND OBSERVATIONS | STRATUM | DYNAMIC CONE PENETROMETER |
|---|---|---|---|---|
| 0.5 | | TOPSOIL | | |
| | | **ELASTIC SILT (MH)**, trace sand and rock fragments, moist, brown | B | |
| | | abundant rock fragments below 2.5 ft | | 16-32 @ 3.0 ft |
| 3.0 | | Hand Auger refusal @ 3.0 ft | | |

omments: Bottom of stacked stone foundation @ 2.5 ft

SCHNABEL ENGINEERING [redacted]

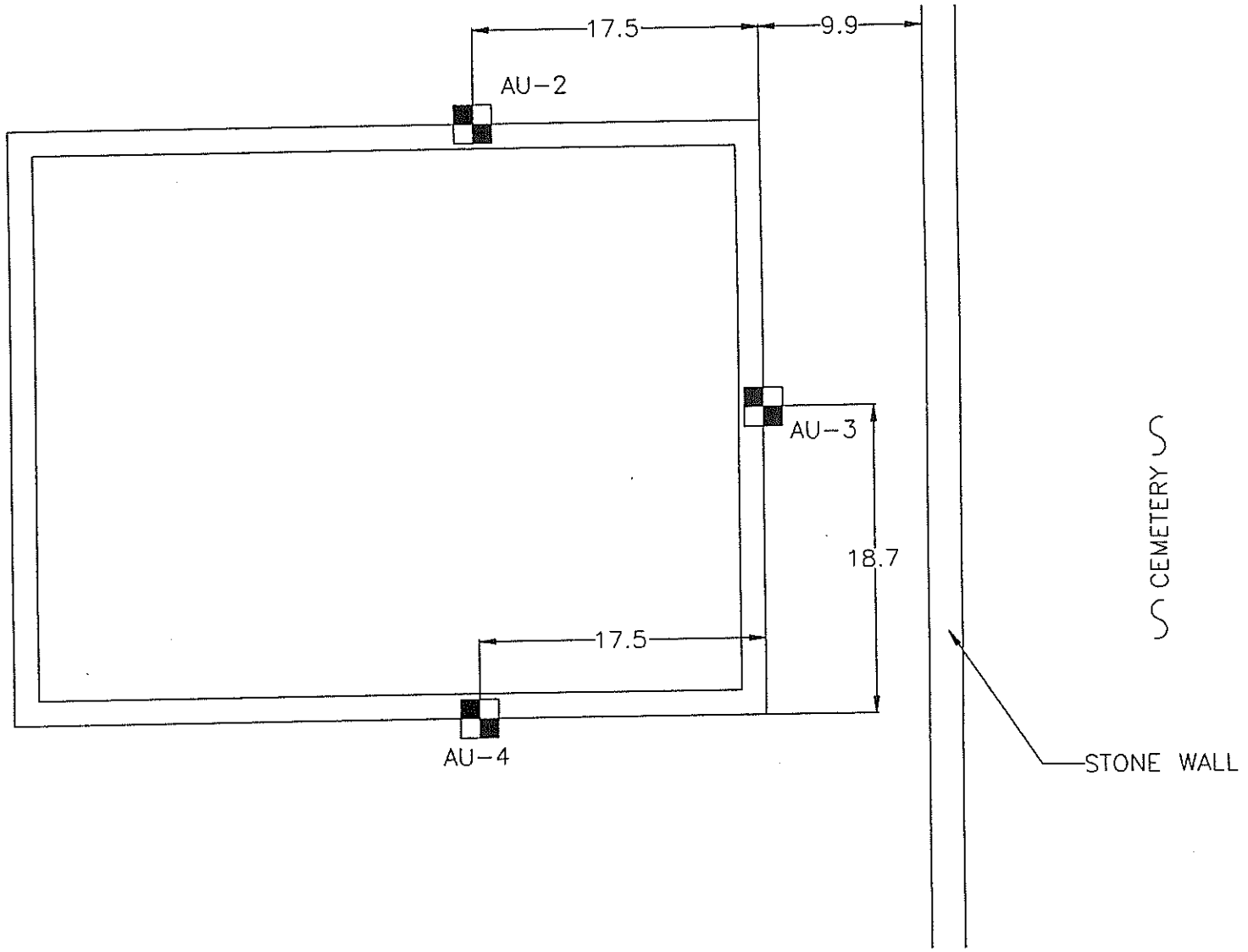
# TEST PIT LOG

CONTRACT NO.: 972717
EXCAVATION NO.: AU-4
PROJECT: MT. ZION OLD SCHOOL BAPTIST SCHOOL
LOCATION: GILBERTS CORNER, VA

DATE STARTED: 11/20/97 DATE ENDED: 11/20/97
SURFACE EL.:
EXCAVATION EQUIP.: HAND EXCAVATED
SEA REPRESENTATIVE: R. DREW THOMAS
GROUNDWATER EL.: NONE ENCOUNTERED

| DEPTH (ft) | ELEV. (ft) | DESCRIPTION OF SOILS AND OBSERVATIONS | STRATUM | DYNAMIC CONE PENETROMETER |
|---|---|---|---|---|
| 0.5 | | TOPSOIL | | |
| | | **ELASTIC SILT (MH)**, with rock fragments, trace sand and organics, moist, brown | B | 8-10-12 @ 2.2 ft<br>27-50/1" @ 2.8 ft |
| 3.1 | | Hand Auger refusal @ 3.1 ft | | |

Comments: Bottom of stacked stone foundation @ 2.5 ft.




L E G E N D

AU-4 TEST PIT LOCATION

SCHNABEL ENGINEERING ASSOCIATES

751 Miller Drive, SE., Suite C-1, Leesburg, Virginia 20175
(703) 779-0773 (703) 589-1246 fax

MT ZION OLD SCHOOL BAPTIST CHURCH
GILBERTS CORNER, VIRGINIA

| TEST PIT LOCATION PLAN | Date: 11-25-97 | Drawn By: RD THOMAS | Contract No.: 972717 |
|---|---|---|---|
| | Scale: 1" = 10' | Checked By: TW LEWIS | Figure No.: 2 |

Appendix B
Contract No. 972717

# SUMMARY OF SOIL LABORATORY TESTING

Soil Laboratory Test Results

# *SUMMARY OF SOIL LABORATORY TEST RESULTS*

*Project:* *Mt. Zion Old School Baptist Church* *Contract No.:* *972717*

*Location:* *Gilberts Corner, Loudoun County, Virginia* *Date:* *4-Dec-97*

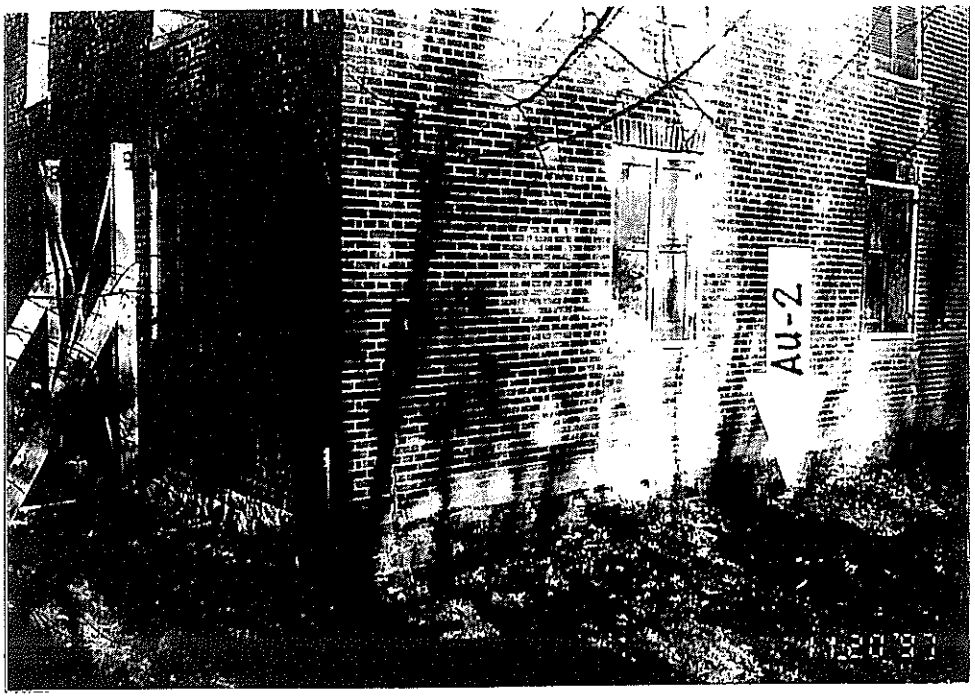
| Boring No. | Depth (ft) | Sample Type | Stratum | Description of Soil Specimen | Sieve Results | | Atterberg Limits | | | Natural Moisture Content (%) | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | Percent Retained No. 4 | Percent Passing No. 200 | LL | PL | PI | | |
| AU-2 | 3.0 | BAG | B | FAT CLAY (CH), with sand, browm | 0.6 | 77.5 | 71 | 33 | 38 | 36.9 | |
| AU-2 | 3.6 | BAG | B | sandy LEAN CLAY (CL), trace rock fragments, brown | 2.5 | 55.6 | 46 | 27 | 19 | 24.4 | |
| AU-3 | 2.5 | BAG | B | ELASTIC SILT (MH), trace sand and rock fragments, brown | 1.8 | 88.1 | 58 | 33 | 25 | 30.5 | |
| AU-4 | 2.8 | BAG | B | ELASTIC SILT (MH), with rock fragments, trace sand, brown | 16.3 | 67.8 | 78 | 36 | 42 | 33.3 | |
| | | | | | | | | | | | |

Notes:

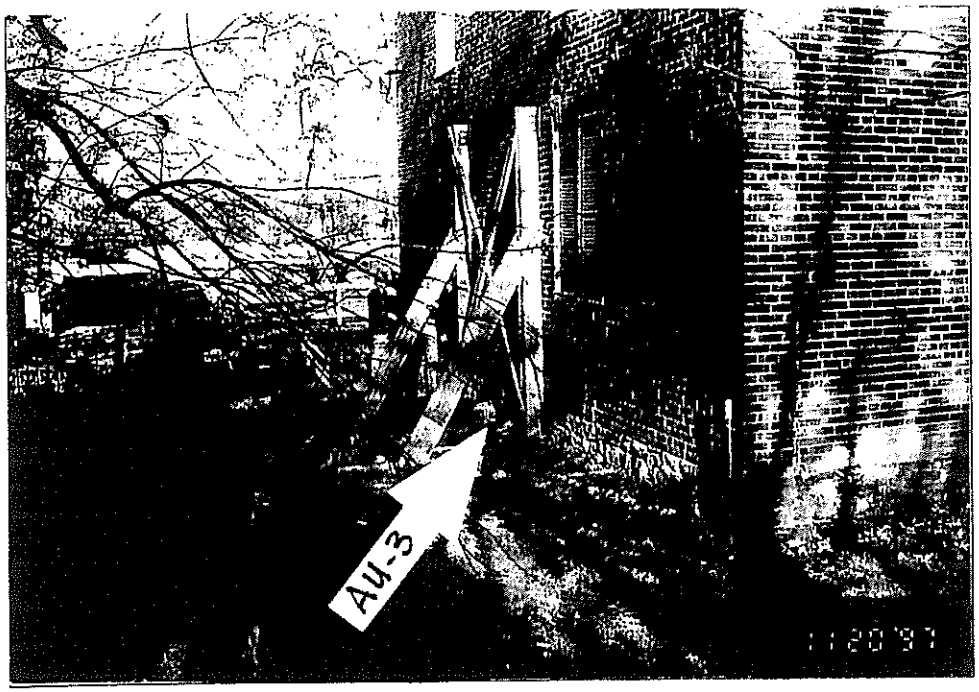
1. Soil tests are in accordance with applicable ASTM standards.
2. Soil classification symbols are in accordance with Unified Soil Classification System, based on testing indicated and visual classification.
3. Visual identification of samples is in accordance with the system used by this firm.
4. Key to abbreviations: LL = Liquid Limit, PL = Plastic Limit, PI = Plasticity Index, NP = Nonplastic

Appendix C
Contract No. 972717

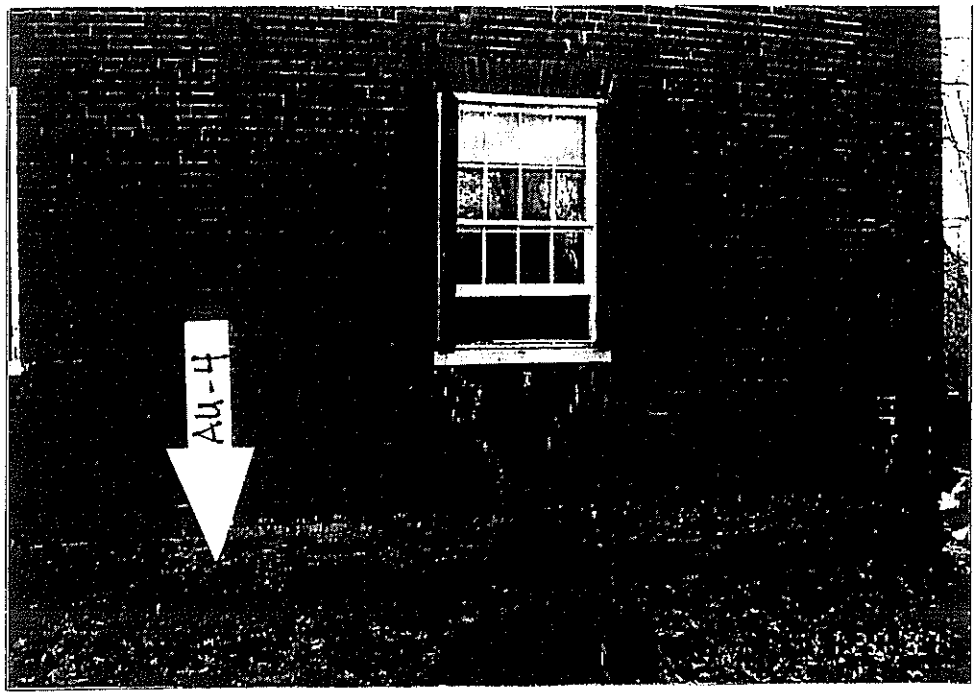
## SITE PHOTOGRAPHS



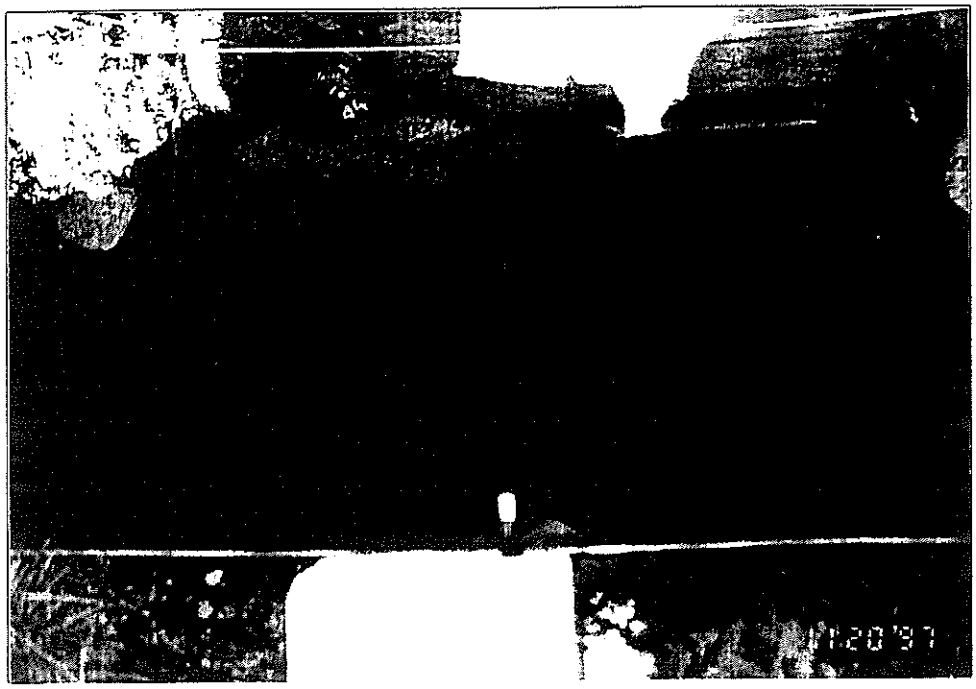

SOUTH WALL – ARROW POINTING AT AU–2



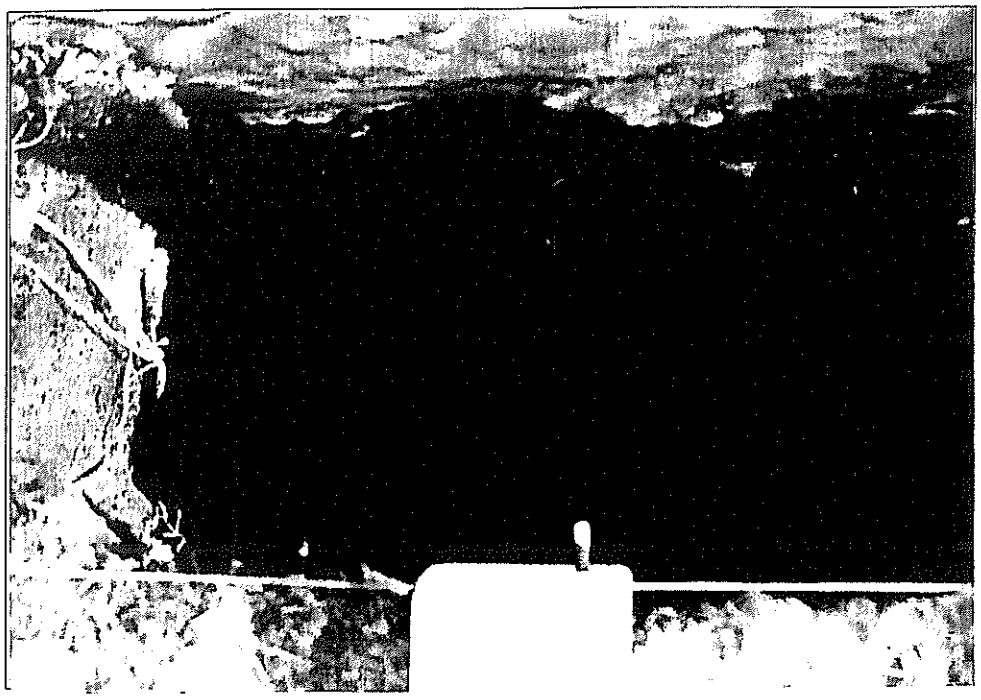

WEST WALL – ARROW POINTING AT AU–3




NORTH WALL – ARROW POINTING AT AU–4



TEST PIT NO. AU–2



TEST PIT NO. AU-3

TEST PIT NO. AU-4

Schnabel Engineering

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March 31, 1998

Mr. Baird M. Smith, AIA
Quinn Evans/Architects
1214 Twenty-Eighth Street, N.W.
Washington, D.C. 20007

Subject: **Geotechnical Engineering Report Addendum, Mt. Zion Old School Baptist Church Damage Study, Gilberts Corner, Loudoun County, Virginia (Our 972717A)**

Dear Mr. Smith:

Schnabel Engineering Associates, Inc. (SEA) is pleased to present this letter report addendum to our Geotechnical Engineering report for the above referenced project dated December 17, 1997. These services have been provided in accordance with our proposal/agreement dated March 2, 1998. This letter presents the results of eight hand auger borings conducted on site and laboratory testing of samples recovered during the hand auger investigation.

## Field Work

On March 9, 1998, Mr. Mike D'Albenzio, Staff Geologist from this office traveled to the Mt. Zion Church site to conduct seven hand auger borings. The hand auger borings were located in the areas surrounding the Church structure that were not investigated during our November, 1997 investigation. An eighth hand auger boring was completed on March 11, 1998 by Mr. R. Drew Thomas, Project Geologist from this office for the purposes of obtaining an undisturbed shelby tube sample of the bearing stratum on the west end of the church structure. Hand auger boring logs and a Subsurface Investigation Plan are presented as Attachment 1.

## Subsurface Conditions and Laboratory Testing

In general, the hand auger borings indicated a subsurface profile consisting of ELASTIC SILT and FAT CLAY soils with varying amounts of sand to depths ranging from 1.6 to 3.5 feet. Underlying these soils is a layer of silty SAND soils which grade into rock at depths greater than the depths of the hand auger borings. Hand auger refusal was encountered at depths ranging from 2.5 to 5.0 feet. Ground water was indicated in hand auger boring Nos. HA-3 and HA-7 at depths of 1.9 and 1.2 feet, respectively.

An undisturbed Shelby tube sample was collected from hand auger boring No. HA-8 from a depth of 2.0 to 3.0 feet below the ground surface. This sample interval was chosen to provide a sample that is representative of the soils upon which the church structure is founded. The Shelby tube sample was submitted to our soil mechanics laboratory for swell pressure testing. In addition, two bag samples of Stratum B soils were analyzed for general identification, natural moisture contents and Atterberg limits. Results of the soil laboratory tests conducted on samples obtained from the field investigation are presented in Attachment 2. Results of Natural Moisture content testing are shown on the hand auger boring logs in Attachment 1.

A bag sample collected from hand auger boring No. HA-3 at a depth of 2.5 feet consisted of FAT CLAY with 84.7 percent fines passing the U.S. Standard No. 200 sieve. This sample had a liquid limit of 71 and a plasticity index of 41 and contained 33.0 percent moisture. This sample classified as CH per ASTM D-2487.

A bag sample collected from hand auger boring No. HA-4 at a depth of 2.5 feet consisted of FAT CLAY with 83.9 percent fines passing the U.S. Standard No. 200 sieve. This sample had a liquid limit of 66 and a plasticity index of 37 and contained 35.1 percent moisture. This sample classified as CH per ASTM D-2487.

An undisturbed Shelby tube sample collected from hand auger boring No. HA-8 from a depth of 2.0 to 3.0 feet was visually classified as LEAN CLAY with sand. This sample was submitted to our soil mechanics laboratory for swell pressure testing per ASTM D-4546, Method B. The results of swell pressure testing indicated a swell of approximately 0.0032 inches after approximately 7.64 days of saturation. This indicates a swell of 0.56 %.

The results of soil laboratory tests completed during the original subsurface investigation and laboratory tests completed for this study indicate subsurface conditions are relatively consistent throughout the perimeter of the building. In addition, swell pressure testing on a soil sample obtained in the area of the observed damage did not indicate significant swell potential. Based on the results of the investigations completed, the most probable geotechnical cause of the observed damage is moisture variation of relatively plastic foundation soils and/or frost heave of soil within the stone foundation elements. During our investigation, we identified the presence of soil particles within the stone wall foundation elements above frost depth of 2 feet. If exposed to moisture, this material may heave. However, it is possible that geotechnical related contributions to the damage may be relatively minor, and other factors not related to soil conditions may be more significant. The remedial measures recommended in our December 9, 1997 report may have little effect on limiting further damage. It may be necessary to rehabilitate stone wall foundation elements to remove soil within the voids between adjacent elements, and to apply some coating to the stone wall to limit migration of soil particles into the rock.

We have prepared this letter report in accordance with generally accepted geotechnical engineering practices. No warranties, either expressed or implied, are made as to the professional advice included in this report. This letter report has not been prepared for use by parties other than those named, or for uses other than those enumerated above. It may not contain sufficient information for purposes of other parties.

We appreciate the opportunity to have been of further service on this project. Please contact the undersigned if there are any questions regarding this report addendum.

Sincerely,

SCHNABEL ENGINEERING ASSOCIATES, INC.

R. Drew Thomas, P.G.
Project Geologist

COMMONWEALTH OF VIRGINIA
RONALD DREW THOMAS
NO. 1320
CERTIFIED PROFESSIONAL GEOLOGIST

Tadeusz W. Lewis, P.E.
Senior Associate

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Attachments:
1) Hand Auger Boring Logs and Location Plan (9 Sheets)
2) Soil Laboratory Test Data (2 Sheets)

SCHNABEL
ENGINEERING ASSOCIATES

# HAND AUGER LOG

**CONTRACT NO.:** 972717A
**DATE STARTED:** 3-9-98 **DATE ENDED:** 3-9-98

**EXCAVATION NO:** HA-1
**SURFACE EL:** ± **GROUNDWATER EL:** ±

**PROJECT:** Mt. Zion Church
**EXCAVATION EQUIPMENT:**

**LOCATION:** SEE BORING LOCATION PLAN
**SEA REPRESENTATIVE:** Michael D'Albenzio

| DEPTH (ft) | ELEVATION (ft) | DESCRIPTION OF SOILS & OBSERVATIONS | STRATUM | GEOSTICK PENETRATION |
|---|---|---|---|---|
| .4 | -.4 | topsoil<br>ELASTIC SILT (MH), roots, moist, brown | B | |
| 1.6 | -1.6 | silty SAND (SM), moist, light brown | C | |
| 3.3 | -3.3 | Hand Auger Refusal @ 3.3 feet | | |

Comments:

SCHNABEL
ENGINEERING ASSOCIATES

# HAND AUGER LOG

CONTRACT NO.: 972717A

EXCAVATION NO: HA-2

PROJECT: Mt. Zion Church

LOCATION: SEE BORING LOCATION PLAN

DATE STARTED: 3-9-98 DATE ENDED: 3-9-98

SURFACE EL: ± GROUNDWATER EL: ±

EXCAVATION EQUIPMENT:

SEA REPRESENTATIVE: Michael D'Albenzio

| DEPTH (ft) | ELEVATION (ft) | DESCRIPTION OF SOILS & OBSERVATIONS | STRATUM | GEOSTICK PENETRATION |
|---|---|---|---|---|
| .3 | -.3 | topsoil<br>ELASTIC SILT (MH), with sand, roots, trace gravel, moist, brown<br>ground water @ 1.0 feet upon completion | B | |
| 1.8 | -1.8 | silty SAND (SM), moist, light brown | C | |
| 2.7 | -2.7 | Hand Auger Refusal @ 2.7 feet | | |

Comments:

SCHNABEL
ENGINEERING ASSOCIATES

# HAND AUGER LOG

**CONTRACT NO.:** 972717A
**DATE STARTED:** 3-9-98 **DATE ENDED:** 3-9-98

**EXCAVATION NO:** HA-3
**SURFACE EL:** ± **GROUNDWATER EL:** ±

**PROJECT:** Mt. Zion Church
**EXCAVATION EQUIPMENT:**

**LOCATION:** SEE BORING LOCATION PLAN
**SEA REPRESENTATIVE:** Michael D'Albenzio

| DEPTH (ft) | ELEVATION (ft) | DESCRIPTION OF SOILS & OBSERVATIONS | STRATUM | GEOSTICK PENETRATION |
|---|---|---|---|---|
| | | FAT CLAY (CH), some organics, moist, light brown | B | |
| | | ground water @ 1.9 feet upon completion | | |
| 3.2 | -3.2 | silty SAND (SM), moist, light brown | C | |
| 4.7 | -4.7 | Hand Auger Refusal @ 4.7 feet | | |

Comments:

SCHNABEL
ENGINEERING ASSOCIATES

[redacted]

# HAND AUGER LOG

**CONTRACT NO.:** 972717A
**DATE STARTED:** 3-9-98 **DATE ENDED:** 3-9-98

**EXCAVATION NO:** HA-4
**SURFACE EL:** ± **GROUNDWATER EL:** ±

**PROJECT:** Mt. Zion Church
**EXCAVATION EQUIPMENT:**

**LOCATION:** SEE BORING LOCATION PLAN
**SEA REPRESENTATIVE:** Michael D'Albenzio

| DEPTH (ft) | ELEVATION (ft) | DESCRIPTION OF SOILS & OBSERVATIONS | STRATUM | GEOSTICK PENETRATION |
|---|---|---|---|---|
| | | ELASTIC SILT (MH), with sand, gravel and roots, moist, light brown | B | |
| 2.5 | -2.5 | FAT CLAY (CH), with sand, moist, grayish-brown | | |
| 3.5 | -3.5 | silty SAND (SM), moist, light brown | C | |
| 5.0 | -5.0 | Hand Auger Refusal @ 5.0 feet | | |

Comments:

SCHNABEL
ENGINEERING ASSOCIATES

[redacted]

# HAND AUGER LOG

**CONTRACT NO.:** 972717A
**DATE STARTED:** 3-9-98 **DATE ENDED:** 3-9-98

**EXCAVATION NO:** HA-5
**SURFACE EL:** ± **GROUNDWATER EL:** ±

**PROJECT:** Mt. Zion Church
**EXCAVATION EQUIPMENT:**

**LOCATION:** SEE BORING LOCATION PLAN
**SEA REPRESENTATIVE:** Michael D'Albenzio

| DEPTH (ft) | ELEVATION (ft) | DESCRIPTION OF SOILS & OBSERVATIONS | STRATUM | GEOSTICK PENETRATION |
|---|---|---|---|---|
| | | ELASTIC SILT (MH), gravel, roots, moist, brown | B | |
| 2.0 | -2.0 | FAT CLAY (CH), with sand, moist, grayish-brown, stiff | | |
| 2.5 | -2.5 | Hand Auger Refusal @ 2.5 feet | | |

Comments:

SCHNABEL
ENGINEERING ASSOCIATES

# HAND AUGER LOG

**CONTRACT NO.:** 972717A
**DATE STARTED:** 3-9-98 **DATE ENDED:** 3-9-98

**EXCAVATION NO:** HA-6
**SURFACE EL:** ± **GROUNDWATER EL:** ±

**PROJECT:** Mt. Zion Church
**EXCAVATION EQUIPMENT:**

**LOCATION:** SEE BORING LOCATION PLAN
**SEA REPRESENTATIVE:** Michael D'Albenzio

| DEPTH (ft) | ELEVATION (ft) | DESCRIPTION OF SOILS & OBSERVATIONS | STRATUM | GEOSTICK PENETRATION |
|---|---|---|---|---|
| .2 | -.2 | topsoil<br>FAT CLAY (CH), roots, trace sand, moist, brown | B | |
| 2.4 | -2.4 | silty SAND (SM), moist, yellowish-brown | C | |
| 3.6 | -3.6 | Hand Auger Refusal @ 3.6 feet | | |

Comments:

SCHNABEL ENGINEERING ASSOCIATES [redacted]

# HAND AUGER LOG

**CONTRACT NO.:** 972717A

**DATE STARTED:** 3-9-98 **DATE ENDED:** 3-9-98

**EXCAVATION NO:** HA-7

**SURFACE EL:** ± **GROUNDWATER EL:** ±

**PROJECT:** Mt. Zion Church

**EXCAVATION EQUIPMENT:**

**LOCATION:** SEE BORING LOCATION PLAN

**SEA REPRESENTATIVE:** Michael D'Albenzio

| DEPTH (ft) | ELEVATION (ft) | DESCRIPTION OF SOILS & OBSERVATIONS | STRATUM | GEOSTICK PENETRATION |
|---|---|---|---|---|
| .2 | -.2 | topsoil | | |
| | | FAT CLAY (CH), trace roots and sand, moist, reddish-brown | | |
| | | ground water @ 1.2 feet upon completion | B | |
| 2.6 | -2.6 | silty SAND (SM), moist, yellowish-brown | C | |
| 3.5 | -3.5 | Hand Auger Refusal @ 3.5 feet | | |

Comments:

SCHNABEL ENGINEERING ASSOCIATES

# HAND AUGER LOG

**CONTRACT NO.:** 972717A
**DATE STARTED:** 3-10-98 **DATE ENDED:** 3-10-98

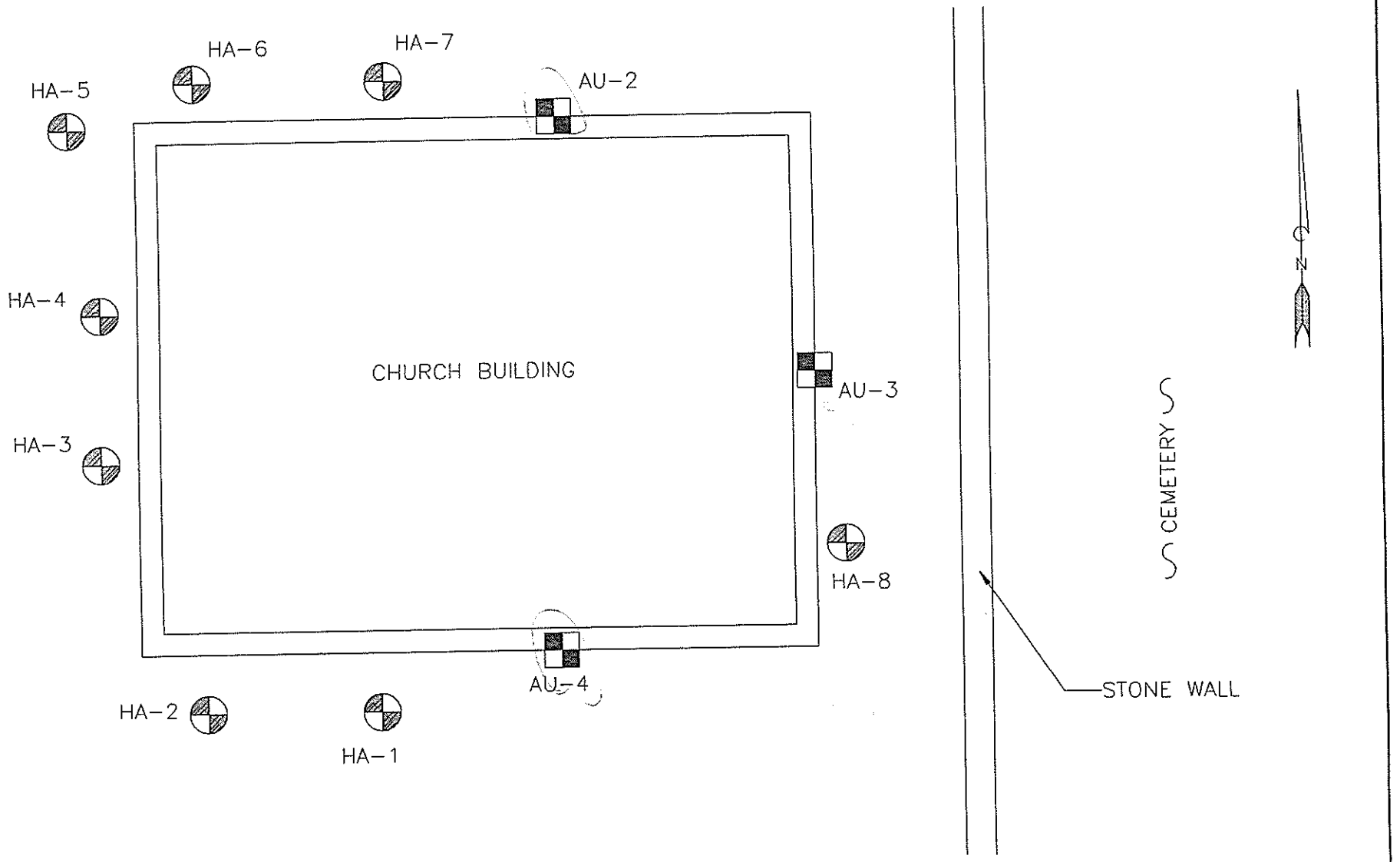
**EXCAVATION NO:** HA-8
**SURFACE EL:** ± **GROUNDWATER EL:** ±

**PROJECT:** Mt. Zion Church
**EXCAVATION EQUIPMENT:**

**LOCATION:** SEE BORING LOCATION PLAN
**SEA REPRESENTATIVE:** R. Drew Thomas

| DEPTH (ft) | ELEVATION (ft) | DESCRIPTION OF SOILS & OBSERVATIONS | STRATUM | GEOSTICK PENETRATION |
|---|---|---|---|---|
| | | lean clay FILL, trace gravel, moist, orange-brown | A | |
| 0.6 | -0.6 | topsoil | | |
| | | ELASTIC SILT (MH), trace sand, moist, brown | B | |
| 2.0 | -2.0 | Hand Auger Refusal @ 2.0 feet | | |

Comments: 3 inch diameter shelby tube driven from 2.0 to 3.0 feet
(12 inch recovery)




L E G E N D

TEST PIT LOCATION

HAND AUGER LOCATION

SCHNABEL ENGINEERING ASSOCIATES
751 Miller Drive, SE, Suite C-1, Leesburg, Virginia 20175
(703) 779-0773 (703) 589-1246 fax

MT ZION OLD SCHOOL BAPTIST CHURCH
GILBERTS CORNER, VIRGINIA

| | | | |
|---|---|---|---|
| SUBSURFACE INVESTIGATION PLAN | Date: 3-11-98 | Drawn By: RD THOMAS | Contract No.: 972717 A |
| | Scale: 1" = 10' | Checked By: TW LEWIS | Figure No.: 2 |

# SUMMARY OF SOIL LABORATORY TEST RESULTS

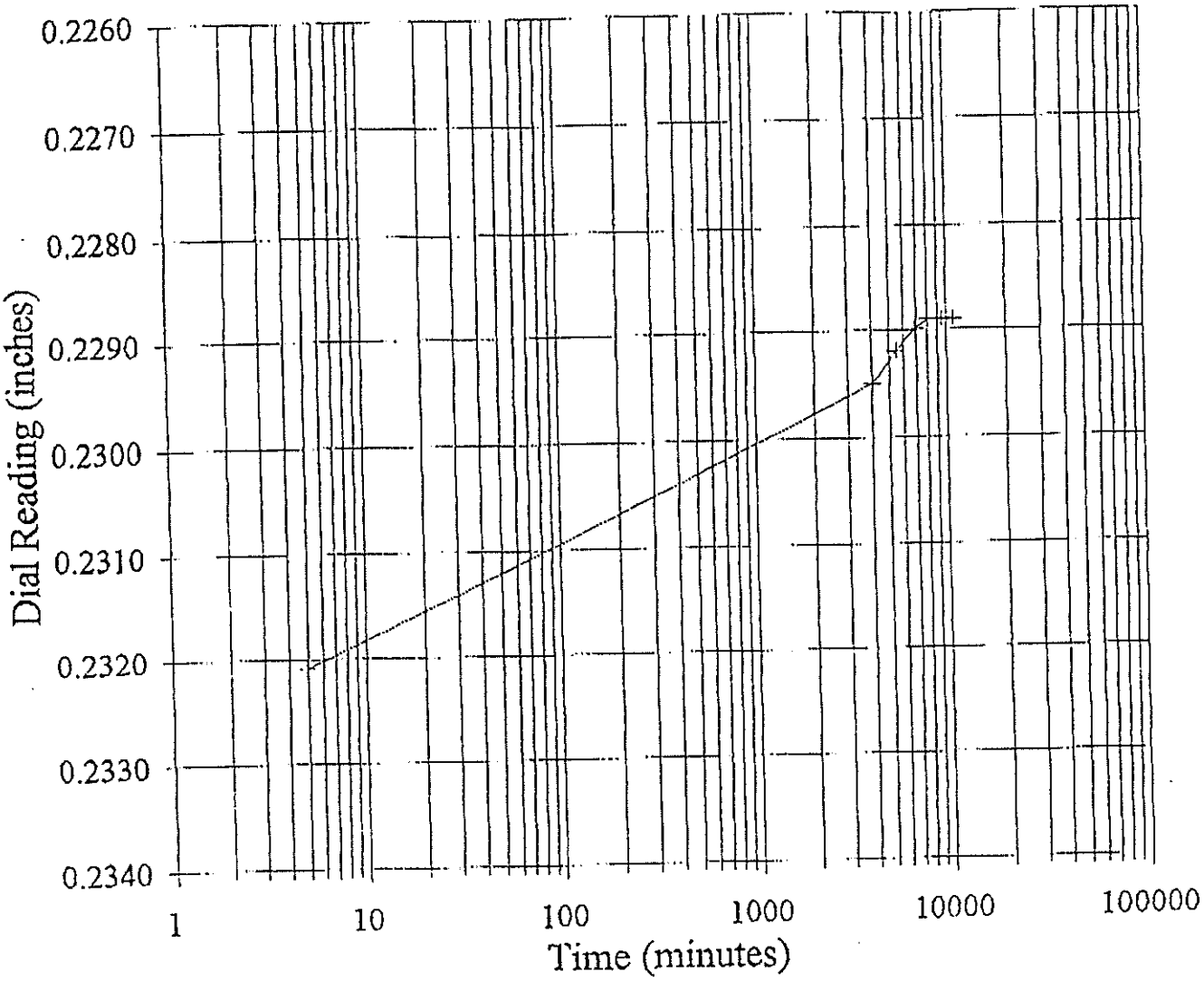
*Project:* Mt. Zion Old School Church
*Location:* Gilberts Corner, Loudoun County, Virginia

*Contract No.:* 972717A
*Date:* 20-Mar-98

| Boring No. | Depth (ft) | Sample Type | Stratum | Description of Soil Specimen | Sieve Results | | Atterberg Limits | | | Natural Moisture Content (%) | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | Percent Retained No. 4 | Percent Passing No. 200 | LL | PL | PI | | |
| HA-3 | 2.5 | BAG | B | FAT CLAY (CH), with sand, brown | 0.3 | 84.7 | 71 | 30 | 41 | 33 | |
| HA-4 | 2.5 | BAG | B | FAT CLAY (CH), with sand, brown | 0 | 83.9 | 66 | 29 | 37 | 35.1 | |
| | | | | | | | | | | | |

Notes:

1. Soil tests are in accordance with applicable ASTM standards.
2. Soil classification symbols are in accordance with Unified Soil Classification System, based on testing indicated and visual certification.
3. Visual identifiaction of samples is in accordance with the system used by this firm.
4. Key to abbreviations: LL = Liquid Limit, PL = Plastic Limit, PI = Plasticity Index, NP = Nonplastic




Sample Description:
LEAN CLAY, with sand, light brown

Classification: CL

SCHNABEL ENGINEERING ASSOCIATES
1600 North Calvert Street, Baltimore, MD 21202
ONE-DIMENSIONAL CONSOLIDATION
TIME vs. COMPRESSION CURVES

| BORING NO: | HA-8 | DEPTH: | 2.0-3.0 |
|---|---|---|---|
| KEY | LOAD (tsf) | PERCENT SWELL | |
| + | 0.03 | 0.5600 | |
| | | | |
| | | | |

Project:
Mt. Zion Church

Contract No.: 972717

JOHN SCOTT
CONSERVATOR OF ART AND ARCHITECTURE

TELEPHONE (212) 714-0620 jscott@panix.com 714-0149 FACSIMILE

NEW YORK CONSERVATION CENTER, INC.
PO BOX 20098LT. NEW YORK, NEW YORK 10011 USA



Analysis Report
Mortar Analysis
Mt. Zion Church
Near Leesburg, VA
QE/A No. 97222

February 20, 1998

Mr. Baird M. Smith, AIA
Quinn, Evans Architects
1214 28th St., NW
Washington, DC 20007

Dear Baird:

Thanks for requesting analysis of the mortar sample listed above. I enclose my standard lab record which shows somewhat more than the following qualities you specified:

****

1. 18 wt% = Presence of lime, (including all acid solubles such as lime and gypsum).

2. None = Presence of a natural cement [Calcium Silicate?] or Portland cement.

3. 3:1 = Approximate ratio of sand to lime.

4. = Approximate grain sizes in digested, washed, sieved sand

| size, mm | sieve | wt % |
|---|---|---|
| > 2.360 | 8 | 00 |
| > 1.180 | 16 | 03 |
| > 0.600 | 30 | 11 |
| > 0.300 | 50 | 34 |
| > 0.150 | 100 | 28 |
| < 0.150 | pan | 24 |

NB: a. fines are 23 wt % of total sample.
b: washed, sieved sand is 59 wt % of total sample

Comments:

The sample received was very small, approximately 9.28 grams, light tan-brown in color, mostly powdered, with several large lumps. One lump appeared to have a dressed and weathered face cornered with a brick interface. [Beyond basic service, this lump was mounted for cross sectional analysis, and an initial examination was done which indicated substantial entrained air voids, as well as sulphation layer just below the weathered surface.]

Stereobinocular microscopical examination (7-40X) of the unaltered lumps revealed scattered open voids, as well as well-distributed white matter in large to very small lumps.The white grains look like lime putty. Most (ca. 90+%) of the aggregate is sand, angular-rounded grains of quartz. SBM examination of washed and sieved sand fractions showed (ca. 1-5%) coal dust probably from lime roasting, as well as a small proportion of fibers, mostly black with adherent black rounded matter. A ca. 13 x .5 x .2 mm fragment of coarse brown wood-pulp paper (?) was observed. SBM examination of undigested pulverized, sieved mortar showed (ca.15%) white grains not present in digested sievings, as well as a low proportion of grains stained green or orange, and some red grains. All grains in undigested sievings are coated with brownish matter; undigested sievings also include sections of the sulphated layer. Grains in digested sievings have less adherent matter.

in undigested sievings →

****

I broke my right index finger badly about the time we spoke about this service, but on receiving your sample, reasonably expected to have the cast off and finish this analysis "on time," this week.

Sincerely,

John Scott

Encl: invoice, lab record

N.Y. Conservation Center

# ARCHITECTURAL CONSERVATION LABORATORY

## MORTAR ANALYSIS

Project/Site: QE/A No. 97222 : Mt Zion Ch. Leesburg VA

Location:

Date sampled:

Analysis performed by: John Scott

Date analyzed: 2/16-19/98

| DESCRIPTION OF SAMPLE | SAMPLE No. |
|---|---|
| Type/Location: Mortar (brown, soft), exterior brickwork | |
| Surface appearance: brown, very porous, | |
| Cross section: sulphated layer near external surface; voids | |
| Color: brown-tan | Texture: porous, matte |
| Hardness: soft / crumbles | Gross Wgt: 9.28g |

COMPONENTS ( 6.54g portion)

| | | | |
|---|---|---|---|
| Fines: | Color: brown-tan | Wgt: 1.5g | Wgt %: 23 |
| | Organic Matter: scattered very fine fibers | | |
| | Composition: | | |
| Acid soluble fraction: | Wgt: 1.16g | | Wgt %: 18 |
| | Desc. of reaction: (15% HCl) immediate & vigorous | | Filtrate color: yellow-brown |
| | Composition: lime and gypsum | | |
| Aggregate: sand washed, after digestion and fines removal | Color: brown-tan-red | Wgt: 3.88g | Wgt %: 59 |
| | Grain shape: mostly angular-rounded | | |
| | Minerology: 90+% silica (quartz) / ≤ 5% (coal (?)) black / ≤ 1% white ≤ 1% mica | | |

Sieve analysis:

| | Screen | % Retained |
|---|---|---|
| >2.360mm | 8 | 0 |
| >1.180 | 16 | 3 |
| >.600 | 30 | 11 |
| >.300 | 50 | 34 |
| >.150 | 100 | 28 |
| | pan <.150 | 24 |

ASSESSMENT

Mortar type: Lime and sand w/ lots of "clay" very fine

Fines:Acid Soluble:Aggregate: 1.3 : 1 : 3

ORTEGA
CONSULTING

Structural Engineering
Building Diagnostics
Architectural Conservation

Richard I. Ortega, P.E.

14 January 1998

Baird M. Smith, AIA
QUINN / EVANS ARCHITECTS
1214 Twenty-Eighth Street, NW
Washington, D.C. 20007

RE: Site Visit 20 November 1997
Mount Zion Old School Baptist Church, US Rte. 50, Aldie Vicinity, Loudoun Cty, VA.
Ortega Consulting Project No. 97-23A

**SITE VISIT REPORT**

Richard Ortega, P.E., R.A., with Mr. Baird M. Smith, of QUINN / EVANS ARCHITECTS, and Mr. R. Drew Thomas, of Schnabel Engineering Associates, conducted a site visit on 20 November 1997. The site visit was performed as a consulting service to QUINN / EVANS ARCHITECTS as part of their current contract with Loudoun County, Virginia, to provide architectural and engineering services for the Mt. Zion Church stabilization project.

The purpose of the site visit was to conduct a visual condition survey of the existing building with particular emphasis on the foundations, roof framing and floor framing. In addition, we observed test pits prepared by Schnabel Engineering. The purpose of this site visit report is to document observations of the existing conditions, and to provide preliminary recommendations regarding additional investigations, or remedies for distressed elements noted in the survey.

The survey was conducted over a single four-and-one-half-hour period. We removed no fabric, nor did we do any sampling or laboratory testing of materials. Visual observations were conducted using available light augmented by hand-held flashlights; examination of suspicious areas was augmented by probing with hand tools such as scratch awls. A fiber optic borescope was used to make limited observations of the first floor framing through holes in the flooring. The following report describes our observations, including preliminary conclusions, and recommendations.

## DESCRIPTION

As described in our report of 11 December 1996, the Mount Zion Church is situated on a relatively flat site, approximately 150' south of U.S. Route 50 (referred to in other references as either the John S. Mosby Highway, or the Lee-Jackson Memorial Highway) approximately 0.8 miles east of its intersection with U.S. Route 15 (the James Madison Highway). Immediately to the west of the building is a cemetery enclosed by a masonry wall approximately 10 feet from the west wall of the church. A board fence defines the other three sides of the church yard.

The building is a two-story, gable-roofed, brick masonry building with stone foundations. It is a simple rectangular structure, approximately 46' by 36' in plan, with gable walls at the east and west, and side walls divided into three bays by pairs of stacked windows and doors. The two principal entrances are in the east gable wall and secondary entrances, giving direct access to the balcony, are in the east bays of the north and south sides. In the interior a "U"-shaped balcony runs along the north, east, and south sides of the sanctuary and there is a raised dais on the west gable wall, which also has two stacked pairs of windows flanking the dais.

The three test pits excavated on the north, south and west sides by the archeologist and geotechnical engineer revealed that the foundations extend uniformly to a depth of approximately 2.5 feet below the existing grade. The walls are of locally obtained rubble stone, which, except for an occasional large stone, are of small dimension (6 to 12 inches), irregularly shaped and not dressed. The mortar used in the walls appears to be a soft lime mortar with low bond strength. The underlying soil below a thin layer of topsoil appeared to be a clay with no apparent rock outcroppings, cobbles, or boulders encountered at the test pits.

The first floor framing appears to be nominal 2x8 joists at 16" on center spanning in the north-south direction over a shallow crawl space. The joists bear on the exterior walls and two lines of intermediate support that are either masonry walls or wood beams supported on stone piers (our limited field of view through the borescope made it difficult to discern the extent of the masonry supports in the interior of the crawl space) dividing the joist spans into three equal bays of approximately 11 feet each.

As we surmised in our earlier site visit, the roof framing consists of four timber Queenpost trusses spaced at approximately 9 feet on center that span in the transverse (north-south) direction and bear on the exterior brick masonry walls. Spanning longitudinally (east-west) between the trusses and bearing on the gable walls, are two rows of timber purlins that support the roof rafters. All

the truss members are nominal 8x8 timbers except the bottom chords which are nominal 8x12 timbers; the purlins are nominal 6x8 timbers laid on the flat. All the timber connections are mortice-and-tenon with no apparent use of metal straps or bolts.

2x6½ ceiling joists spaced at 16 inches on center span between the bottom chords of the trusses with the end bays bearing on the gable walls. The joists are flush-framed into the trusses with mortise-and-tenon connections; The tenons are full width, about one inch thick, of unknown depth, and probably not pegged as we saw no treenails in the top surface of the truss chord. The last joists parallel to the north and south walls support ladder framing that is mortised into the joist and spans across the brick masonry walls. A wood plate laid across the tops of the ladder framing joists provides the bottom bearing for the roof rafters.

2x roof rafters of depths varying from 3½ to 5 inches spaced at approximately 26 inches on center span, in matched pairs morticed at the ridge, north-south, bearing on the purlins and the exterior walls.

Although we took no wood samples for identification, the heavy timbers appear to be hewn from a soft wood, perhaps pine, and the joist and rafters are a sawn hardwood, perhaps oak.

## OBSERVATIONS

### Foundations and Soil Conditions

As described above, the foundations are shallow rubble stone masonry walls, 16 to 20 inches thick, about 30 inches deep and laid using a soft lime mortar with negligible bond strength. The test pits exposed little additional evidence of distress in the foundations other than what was visible above grade. That is, the cracks, subsidence, or differential movement visible above grade continued below grade, but there was no evidence of defects in the construction of the foundation that could be considered the sources of the distress evident in the masonry walls above. Aside from loss of mortar, we saw no bulges or displacement of the walls that would indicate that they were failing. It was noted that the stone masonry of the south foundation looked slightly different from that of the north foundation wall, but this may be merely an indication of the difference in technique of two different masons working on the same job and appears to have had no effect on the performance of the foundations

A visual inspection of the soil gave no indication that the soil might be incompetent to support the loads imposed by the structure. At the time of the site visit, Drew Thomas saw nothing unusual, or wrong, with the soil that might indicate that its performance was the source of the building distress. Subsequent laboratory analyses by Schnable Engineering Associates, Inc., indicate that at least some of the underlying clay stratum might be subject to excessive swelling and shrinkage with fluctuations in moisture content (see their *Geotechnical Engineering Report, Mt. Zion Old School Baptist Church, Damage Study, Gilberts Corner, Loudoun County, Virginia*, dated December 9, 1997).

**Brick Masonry Walls**

As note in our site visit report of 11 December 1996, the exterior brick masonry walls, especially at the west end, exhibit significant distortion and distress. Since that report, two timber raking shores have been built against the west gable wall to prevent further movement. A review of photographs taken in 1996 indicate that there has been no discernible addition distress in the walls. We noted that the raking shores were not in full contact with the west gable wall. Either they were never in contact, or the wall shifted eastward, or the shoring moved, but in any case, the gap would indicate that the outward bulging of the wall does not appear to have worsened since the shore was put in.

Part of the problem with the failed jack arches is attributable to the overall distress and movement of the brick masonry walls at, or near, the window openings. Because the window openings are what are called "stress raisers" if there is subsidence or any other type of differential movement of the wall, the distress will express itself around window openings. Once the movement occurs, the compressive force that kept the jack arch in place is lost, the arch is subjected to tensile forces, and the arch collapses; this is part of what has happened, especially at the west end of the building.

It should be noted that while jack arches are stylistically appropriate for buildings of certain periods, they are notoriously poor performers as lintels. Furthermore, if the ones at the Mt. Zion Church were intended to be working jack arches, i.e., support the loads above the windows, as they appear to have been, then the original construction of the arches was flawed:

- A single soldier course jack arch is too shallow for the width of the openings. The line of thrust of the compressive force cannot be maintained within the middle third of the arch, thus, the jack arch goes into tension and fails.

- All arches rely on a substantial gravity load above them to insure constant compressive forces within the arch to maintain the arch's stability. The jack arches over the upper windows at the Mt. Zion Church appear to be very lightly loaded and probably never worked well.

It is likely that the jack arches, whether intentionally or not, have always been largely supported by the window frames. As the window frames have deteriorated, the arches have dropped.

## Floor Framing

Despite very limited access to inspect the floor framing we made two significant observations.

- There are termite tubes visible on the sides of at least two joists on the south side of the building. It could not be determined whether the infestation is active, or from long ago, but as we could see no evidence of repairs in the crawl space, it is unlikely that any damage that resulted from the infestation, whether active or dormant, has been repaired.

- The noticeable floor subsidence at the east gable wall appears to be the result of a loss of joist support and not rot. Apparently there were several shims under the last joist, probably to level it, and they have fallen out, which has left the last joist without support at one end.

## Roof Framing

Although we had full access to the attic space, it should be noted that inspection, especially close inspection, of some of the elements was hindered by the large deposits of bat droppings throughout the space.

The roof framing, especially the rafters, is very light and is probably subjected to stresses well above what would be allowed by modern design standards; nevertheless, we noted no evidence of damage, distress, or failure from overloading, other than deflections that would also be considered excessive by modern standards.

The timber trusses also appear to be light by today's standards, but they were obviously built with care and well conceived. The joinery is nicely crafted and tightly fitted. Unfortunately, every one

of the four trusses has one, or both, failed heel joints, caused by either insect damage, rot, or both. Heel joint failures of timber trusses are relatively in buildings of this period and this manner of construction. In part, it is due to the inherent limitations of the materials and the methods of construction available at the time of construction -- highly stressed joints in timber structures, and the heel joint is the most highly stressed in a truss, are subject to failure due to the long-term weakening, or creep, of the wood between the wood fibers. In part it is due to the fact that the heel joint is usually below the part of the roof most vulnerable to roof leaks -- the gutters, which creates conditions favorable for the formation of rot, or the infestation of wood boring insects, at the most critical joint of the truss.

At the Mt. Zion Church, the heel joints of the trusses have been affected by both rot and insects. Except for the truss nearest the west gable, which exhibits a noticeable dip at the south end, there appears to have been little displacement, or distortion of the trusses. This may be because they are very lightly loaded and there has not been a recent heavy snowfall, or windstorm, to test the residual strength of the trusses.

## CONCLUSIONS AND RECOMMENDATIONS

Heretofore, in my earlier site visit report, and in the previous reports on the building, the item of greatest concern has been the west gable wall. As a result of this latest site visit, structural repairs to the roof and installing a weather tight roof system should take a higher priority.

The distress in the masonry walls does not appear to be progressing at a rapid pace, if at all. The preliminary geotechnical report indicates that the subsurface soil characteristics that are a probable cause of the soil distress are seasonal and cyclic, and may be very difficult, and expensive to remedy. Finally, the existing shoring appears to be adequate to limit any gable wall failure to a localized loss of the wall.

The truss distress, however, appears to be at a critical stage. Failure of any heel joint is likely to be sudden and catastrophic with the potential of shedding loads to adjacent trusses and inducing a generalized roof failure. This is not a cyclic problem, but a constant one, and it is impossible to predict when a live load sufficient to cause failure might be applied to the roof.

There are a number of methods to repair the roof trusses. One can reinforce them with steel plates and bolts, or, if the stresses are low enough they can be reinforced with wood plates and

bolts, which are traditional methods, or they can be reinforced with more modern methods employing glass fiber reinforced polyester rods and epoxies, which are more recently developed methods. Attached to this report you will find sketches of repairs done by this office using a variety of methods.

Considering the dimensional instability of the fat clays with changes in moisture, I am also curious about the effects the large trees that used to be at the west end of the building may have had on the shrinkage of the clay stratum, and why that has not occurred at the east end of the building (see attached "Case of the Cracked Corner").

It will be extremely expensive to underpin the existing rubble masonry foundations, because they are not likely to bridge over anything but a very narrow approach pit without collapsing, so the work would be very slow, and would probably have to be done by hand. Unfortunately, our three test pits did not reveal significantly different conditions around the building, nor do we know why there is no similar distress at the east end of the build. The soil conditions suggest that we need to do more subsurface investigation or monitoring to ascertain the extent of the "fat clay" stratum, the severity of the soil heaving, and the frequency of the "heaving" of the soil.

I would suggest that at the least we should dig some more test pits, certainly one at the east gable. I would also that we might be able to test the swelling of the clays by installing settlement plates in the ground around the building and monitoring then at regular intervals to see if they do move. The monitoring might be further refined by covering the settlement plates and a band of earth about 12 feet wide around one side of the church with a plaster barrier to prevent rainwater from entering the soil, while leaving the other side of the church uncovered. If the monitoring of this experiment yields significant differences between the two sides of the building, then we may have found the principal source of the water and have a potentially easier way to stabilize the foundation conditions *in lieu* of underpinning.

If you have any questions regarding this report, or if I can be of further assistance, please call me.

Sincerely,

Richard I. Ortega, P.E., R.A.
*File: c:\wpdocs\sv_repor\1998\mtzionsv*